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SUBJECT	Review Note RN-0141_Structural Calculation of Diversion Tunnel Lining			

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1	RN-0141	RR	UT1-C-010-CVL-DC-42005	0B	Structural Calculation of Diversion Tunnel Lining	
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UPPER TRISHULI-1 HEP (216MW)			
Client	**Nepal Water and Energy Development Company Private Limited**		
Owner's Engineer	**Tractebel Engineering GmbH / Jade Consult**		
REVIEW NOTE			
Contractor	**Doosan Enerbility**		
Title of the Document	Structural Calculation of Diversion Tunnel Lining		
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1) ACI 318-14 is used for design of lining. Since ACI 318-19 supersedes ACI 318-14, it is requested to use ACI 318-19 for the design purpose.

2) The concrete compressive strength adopted for lining is 24MPa in section 3.1 (1). a. of the report, but ACI 350-06, clause 1.1.1 requires the minimum compressive strength of 28 MPa for hydraulic structures. Besides, the flow velocity is not considered in design of lining strength. As per the discussion with geotechnical expert, it is found that the flow velocity is greater than 10m/s. As per ACI 210R-06, fig. 3.2, it is seen that 10m/s bottom velocity could transport the particle size of around 800mm and greater. The sediment size might not be so big in our case but there is no any calculation regarding the abrasion of concrete cover thickness. The fig. 3.2 from ACI 210R-06 is shown below:

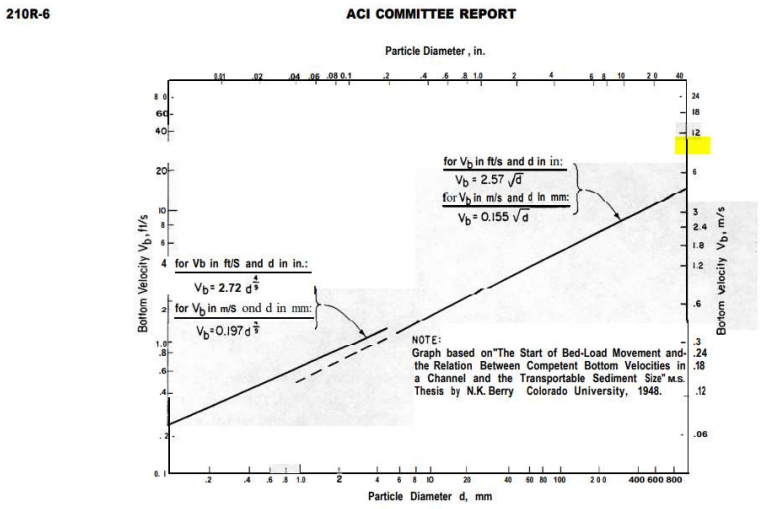

210R-6
ACI COMMITTEE REPORT
Particle Diameter , in.
Bottom Velocity V_b, ft/s
Bottom velocity V_b, m/s
for V_b in ft/s and d in in:
$V_b = 2.57\sqrt{d}$
for V_b in m/s and d in mm:
$V_b = 0.155\sqrt{d}$
for Vb in ft/S and d in in.:
$V_b = 2.72\, d^{\frac{1}{9}}$
for V_b in m/S ond d in mm:
$V_b = 0.197\, d^{\frac{1}{9}}$
NOTE:
Graph based on "The Start of Bed-Load Movement and the Relation Between Competent Bottom Velocities in a Channel and the Transportable Sediment Size" M.S. Thesis by N.K. Berry Colorado University, 1948.
Particle Diameter d, mm

Fig. 3.2-Bottom velocity versus transported sediment size

EM 1110-2-2401 requires the minimum clear cover of 6 in (150mm) for the concrete subjected to

cavitation or abrasion erosion and 4 in (100mm) for foundation. The screenshot from EM 1110-2-2104, Table 2-1 is presented below:

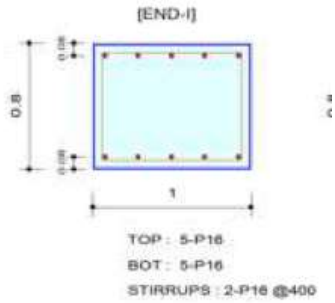
EM 1110-2-2104
30 Nov 16

Table 2-1. Minimum Clear Distance from the Edge of the Reinforcement to the Surface of the Concrete.

Concrete Section	Minimum Clear Cover of Reinforcement (in.)
Unformed surfaces in contact with foundation	4
Formed or screeded surfaces subject to cavitation or abrasion erosion, such as baffle blocks and stilling basin slabs	6
Formed and screeded surfaces such as stilling basin walls, chute spillway slabs, and channel lining slabs on grade:	
Equal to or greater than 24 in. thick	4
Greater than 12 in. and less than 24 in. thick	3
Equal to or less than 12 in. thick	In accordance with ACI 318.
NOTE: In no case shall the cover be less than: 1.5 times the nominal maximum size of aggregate, or 2.5 times the maximum diameter of reinforcement.	

The cover dimension mentioned in the section diagram is unclear and seems less than 100mm. Please mention the clear cover adopted for the design of concrete lining in the design report. The screenshot of section diagram is given below:

2. Section Diagram



An article titled “Field Investigation on Hydroabrasion in High-Speed Sediment-Laden Flows at Sediment Bypass Tunnels” by Michelle Muller-Hagmann, Ismail Albayrak, Christian Auel and Robert M. Boes, 2020 suggest that for 9.8m/s velocity the average depth of wearing for M110 concrete is around 31mm in operational four years. The maximum wearing depth is roughly twice the average wearing depth, that is, about 62mm as per the paper. A screenshot from the paper is given below:

Table 4. Mean annual SBT operation conditions, bedload transport masses and abrasion depths in the Pfaffensprung SBT for the years 2012 to 2015.

Parameter	Unit	2012	2013	2014	2015	Average
SBT operation duration T	[d]	91	61	55	96	76
Mean discharge in SBT $\overline{Q}_{SBT}$	[m^3/s]	35.5	32.6	25.9	34.5	32.1
Mean flow depth $\overline{h}$	[m]	0.82	0.76	0.61	0.80	0.75
Mean flow velocity $\overline{U}$	[m/s]	9.8	9.8	9.7	9.8	9.8
Mean Shield's parameter $\overline{\theta}$	[–]	0.042	0.039	0.035	0.042	0.040
Bedload mass BL	[10^3 to/year]	460	370	140	430	350
Mean Abrasion Depths a_m		**2012**	**2013**	**2014**	**2015**	**2012–2015**
High-strength concrete 1 (C1)	[mm]	15.4	9.3	1.3	5.0	31.0
High-strength concrete 2 (C2)	[mm]	-	8.9	1.5	-	10.4
Urner Granite 1 (G1)	[mm]	2.9	0.6	0.5	1.3	5.3
Urner Granite 2 (G2)	[mm]	-	1.4	0.6	-	2.0

EM 1110-2-2901, clause 6-2. e .(3) also suggests the same thing for the lining requirement. The screenshot from EM 1110-2-2901, clause 6-2. e .(3) is given below:

> (3) In practice, fluid velocities are limited so that turbulent conditions and the possibility of damage to the structure are limited. Velocities of less than about 3 m/s (10 ft/s) are considered safe in tunnels with no lining. Velocities between about 3 and 6 m/s (10-20 ft/s) usually necessitate concrete linings. For velocities greater than 6 m/s (20 ft/s), the risk of cavitation increases, and special precautions like steel or other types of inner lining must be taken to protect the inside of the structure. Where the water will carry sediments (silt, sand, gravel) the velocity should be kept below 3 m/s (10 ft/s).

This suggests the need to high strength concrete lining the tunnel. Some alternative could be high strength material like Granite layer lining, steel lining etc.

So, contractor is requested to investigate/ calculate the lining material strength and adopt the lining material based on the abrasion and cavitation requirement as well. The minimum concrete grade of 28 MPa as per ACI 350-06, clause 1.1.1 will most probably not govern the design.

3) Round bar of 280 MPa is not available in Nepal. Generally 415 MPa or 500 MPa bars are used as shear bars. So, please use the available bars in the design. Screenshot from report is given below:

Figure 4.2-1 Calculation Result of Shear Reinforcement

V(kN)	Nu(kN)	b(m)	h(m)	Ag(m^2)	f'c(MPa)	fy(MPa)	φ	Vc(kN)	Vss(kN)	Reinforcement
145.3	88.4	1.0	0.3	0.3	24	280	0.75	184.5	9.21	R10@400X300

4) Table 3.2-1, shows the normal and tangential spring stiffness of surrounding rock (Unit length). The beam lengths are divided into arc length corresponding to 5-degree central angle, the spring constant for each beam element needs to be calculated based on that length. However, the provided spring stiffness is calculated for 1m length and is about 4 times greater than that required for 5 degree arc length. In addition, the discretization length of the tunnel at the vertical face and invert level are different as seen from the figure 3.2-2. Please provide the calculation for spring constant for those parts as per the length of the element.

The sample calculation for rock class II is shown below:

UT-1

EM 1110-2-2901, formula (9-18)		
Spring Constant Calculation for Rock Class II		
Er =	2.05E+04	MPa
Mean tunnel radius, r =	2.983	m
Poisson's ratio, v =	0.25	
Calculation:		
Tunnel circumference, C =	18.74	m
Angle subtended by each arc element, θ =	0.087	radians
Length of element considered, b (flow dirtn) =	1.00	m
kr =	1431170	kN/m
kt =	572468	kN/m
As per provided report		
kr =	**5857143**	**kN/m**
kt =	**2342857**	**kN/m**

5) Table 3.3-1 considers the grout pressure on the design but the value adopted for the grout pressure is not mentioned in the report. Please provide the grout pressure considered for the design.

6) If excavation is to be carried out by explosive then horizontal force needs to be increased by 30% as per EM 1110-2-2901, Box 9-1. But 3.3.2 (1) does not consider extra 30% load. If the explosives are not to be used, please mention the method of excavation.

7) As per EM 1110-2-2901, cl 9-1h, with proper drainage, the external water pressure shall not be taken less than lesser of 25% of the existing external water pressure and three tunnel diameter due to the possibility of clogging. The value adopted for external pressures seem to be very low as per the report. For example, the external pressure head of class IV rock is 220m (Table 3.3-4). The external water pressure cannot be less than lesser of 0.25*220m = 55m and 3*6 = 18m. But from table 3.7, the adopted value seems to be 8.4m. Screenshot from EM 1110-2-2901, cl 9-1h is given below:

h. Temporary or permanent drainage. It may not be necessary or reasonable to design a lining for external water pressure. During operations, internal pressures in the tunnel are often not very different from the in situ formation water pressure, and leakage quantities are acceptable. However, during construction, inspection, and maintenance, the tunnel must be drained. External water pressure can be reduced or nearly eliminated by providing drainage through the lining. This can be accomplished by installing drain pipes into the rock or by applying filter strips around the lining exterior, leading to drain pipes. Filter strips and drains into the ground usually cannot be maintained; drain collectors in the tunnel should be designed so they can be flushed and cleaned. If groundwater inflows during construction are too large to handle, a grouting program can be instituted to reduce the flow. The lining should be designed to withstand a proportion of the total external water pressure because the drains cannot reduce the pressures to zero, and there is always a chance that some drains will clog. With proper drainage, the design water pressure may be taken as the lesser of 25 percent of the full pressure and a pressure equivalent to a column of water three tunnel diameters high. For construction conditions, a lower design pressure can be chosen.

Besides, the minimum external water pressure as per EM 1110-2-2901 cl 9-1h is 18m in our case. As per EM 1110-2-2901, cl 9-4d it is reasonable to consider external water pressure equal to internal water pressure. The internal water pressure ranges from 16.5m to 28.5m. Hence, the contractor is recommended to adopt the external water pressure value of **higher of 18m and internal pressure at the section**. So please revisit the calculation.

8) The roof water reduction for intersection provided in table 3.7 is not matching with figure 3.3-3. Please revisit the calculation. The screenshots from the report are shown below:

Table 3.7 Calculation of External Water Pressure

Chainage	Rock class	Lining Thickness(m)	Roof water reduction Average value (m) /(kPa)	Side wall water reduction Average value (m)/(kPa)	Invert water reduction Average value (m)/(kPa)
D0+020.000	III	0.5m	8.5/83.3	5.5/53.9	3.0/29.4
D0+195.039 Intersection	II~III	0.8m	60.6/593.88	5.5/53.96	3.0/29.4
D0+226.935	IV	0.5m	8.4/82.32	5.6/54.88	2.9/28.42
D0+385.553	III	0.3m	7.7/75.46	5.4/52.92	2.4/23.52

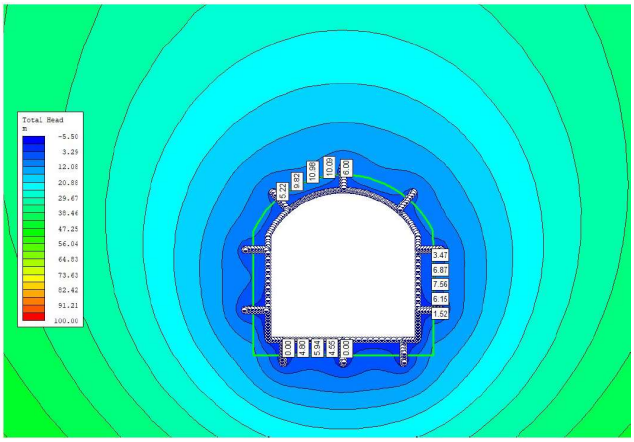


Figure 3.3-3 The Seepage Calculation Result of Intersection Section Chainage 0+020.00

9) The crack width calculation for the lining is not provided in the report. In order to avoid the crack check the load combination shall be multiplied by environmental durability factor (Sd) as per ACI 350-06, clause C.9.2.9.

C.9.2.9 — Required strength ***U*** shall be multiplied by the following environmental durability factors (S_d) in portions of an environmental engineering concrete structure where durability, liquid-tightness, or similar serviceability are considerations. These durability factors shall not be used for prestressed reinforcement or for designs using service loads and permissible service load stresses per the alternate design method in Appendix I. For applicable use of the environmental durability factor (S_d) in conjunction with load combinations that include earthquake loads, see Section 21.2.1.8.

C.9.2.9.1 — Flexural stress: $S_d = 1.3$.

C.9.2.9.2 — Direct tensile stress (including hoop tension): $S_d = 1.65$.

C.9.2.9.3 — Excess shear stress carried by shear reinforcement: $S_d = 1.3$.

10) The minimum reinforcement As,min shall fulfill the requirement of ACI 350-06, Table 7.12.2.1. Though the reinforcement will be checked only after incorporating the comments, it is seen that the reinforcement provided doesnot meet the requirement of minimum requirment in some places. For example, in table 5.1 the distribution bar of 12mm@300mm c/c is provided.

Table 5.1 Calculation Summary of Concrete Lining Reinforcement

Chainage	Rock class	Lining Thickness(m)	Main reinforcement diameter & Spacing (mm)	Distributed reinforcement diameter & spacing (mm)	Shear reinforcement diameter & spacing (mm)
D0+020.000	III	0.5m	D20@200 Doubled-sided	D16@200 Doubled-sided	R10@400X400
D0+195.039	II~III	0.8m	D16@200	D12@300	Roof & side wall D16@400X300

For 0.8m section the minimum bar shall be at least 16mm@200mm c/c in two layers as per ACI 350-06, table 7.12.2.1. The reinforcement quantity might increase depending upon the joint length. The screenshot of ACI 350-06, table 7.12.2.1 is given below:

TABLE 7.12.2.1—MINIMUM SHRINKAGE AND TEMPERATURE REINFORCEMENT

Length between movement joints, m	Minimum shrinkage and temperature reinforcement ratio	
	Grade 280	Grade 420
Less than 6	0.0030	0.0030
6 to less than 10	0.0040	0.0030
10 to less than 12	0.0050	0.0040
12 and greater	0.0060*	0.0050*

*Maximum shrinkage and temperature reinforcement where movement joints are not provided.
Note: This table applies to spacing between expansion joints and full contraction joints. When used with partial contraction joints, the minimum reinforcement ratio shall be determined by multiplying the actual length between partial contraction joints by 1.5.

Concrete sections that are at least 600 mm may have the minimum shrinkage and temperature reinforcement based on a 300 mm concrete layer at each face. The reinforcement in the bottom of base slabs in contact with soil may be reduced to 50 percent of that required in Table 7.12.2.1.

11) Please provide the software files (Phase 2D and Midas Gen files) used in this report, so that it could be verified quickly.

12) With respect to further design of tunnel liners of the waterways at UT-1, a more up-to-date model could me used to verify the results from Midas Gen software. The spring model is certainly from an analytical point of view, correct. However nowadays there a computational models available which provide more insight view of the conducted analyses. The designer is using the PHASE2 code for the seepage analyses, already. With this tool (more) adequate calculations can be conducted for the tunnel liner.

13) The document is returned for resubmission.

DISCLAIMER

Any approval will not release the Contractor from any of his obligations under the Contract.

Dr. Subin Desar (Structural Engineer)